STEEL AND CONCRETE SUBSTRUCTURE
OF A
RIVER CROSSING FOR HIGHWAY TRAFFIC

BY

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ARMOUR INSTITUTE OF TECHNOLOGY

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DESIGN AND ESTIMATE OF COST OF
STEEL SUPERSTRUCTURE AND CONCRETE SUBSTRUCTURE OF A
RIVER CROSSING FOR HIGHWAY TRAFFIC.

A THESIS

Presented by

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Approved A. E. Phillips
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The Problem.

The problem consists of the design of a river crossing for highway traffic for a suburb of Chicago. The location and general conditions are assumed as shown on the blue print. The river is bridged by three spans, the middle span being seventy five feet and the two end spans each sixty feet. There will be two solid concrete piers and two reinforced concrete abutments.

List of Illustrations.

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Design of Middle Span.

Span = 75' - 0".

Weight = 5' - 0"

Width center to center of Trusses = 16' - 6".

Pratt Trusses, parallel chords, no sidewalks.

Asphalt floor with concrete foundation.

Specifications - Illinois Highway Commission.

Class "C" bridge.

$\frac{3}{4}$" rivets.

Design of Floor.

Assume concrete foundation 4" thick. (FIG. 2)

Buckle plates - 5/16" thick.

Binder course - $1\frac{1}{2}$" thick.

Asphalt - 2" thick.

Total weight of floor taken as 160 # per cu.ft.

Weight of floor = 105 # per square foot.

Weight of floor per panel = 15 x 17 x 105 = 26775 #

Heavier wheel load = 4 tons = 8000 #.

Space the stringers 2' - $1\frac{1}{2}$" apart.

Design of Roadway Stringers.

To find the center of gravity:-

$$\frac{8000 \times 10}{12000} = 6.667 \quad (\text{fig. } 3)$$

For the maximum bending moment, the heaviest load must be upon the span and a wheel load must be at the section and this wheel load causing the maximum bending moment must be as far on one side of the center as the center of gravity of all the loads is upon the other. In this case however the maximum bending moment is caused by a wheel load placed at the center of the span.

$\frac{10 - 6.667}{2} = 1.667'$ distance of center of stringer to center of wheel # 1. This makes wheel # 1 come off the span. There put wheel #2 at the center of the span.

$M = \frac{8000}{2} \times 7.5 \times 12 = 360000$ "#

$\frac{360000}{13000} = 27.7$ Section modulus.

Carnegie :- Try 12" I-beam @ 27.5 # S = 33.3

Weight of floor = 105 # per foot.

Weight of stringer = 27.5 # per foot.

Total weight = 132 # per foot. D.L.

$D.L.B.M. = \frac{wl^2}{8} = \frac{132.5 \times 15^2 \times 12}{8} = 44800$ "#

$\frac{44800}{13000} = 3.44$

27.7 + 3.44 = 31.14 O.K. since 33.3 is greater than 31.14

Design of Track Stringers.

Live load is 18 tons on two axles 10 foot centers. (Fig.4)

$$L.L.B.M. = \frac{9000}{2} \times 7.5 \times 12 = 405000 \text{ "\#}$$

$$S = \frac{405000}{13000} = 31.2$$

Try 12" I-beam @ 31.5 # S = 36.0

D.L. 105.0 #

31.5

40.0 T-beam

30.0 Rail

206.5 # Total.

$$M = \frac{wl^2}{8} = \frac{207 \times 15^2 \times 12}{8} = 69800 \text{ "\#}$$

69800 + 361200 = 431000

$$S = \frac{431000}{13000} = 33.2 \quad O.K.$$

(Art.57) $\frac{75}{b}$ is less than 12"

Design of Floor Beams.

Assume concentrated loads are supported by two stringers, the maximum shear comes when

one axle at the end of the panel.

Weight of the concrete floor = [illegible] x 15 x 17.5[illegible]

= 27250 [illegible]

[illegible]-12" I-beams [illegible] x 15 x [illegible] = [illegible]

7-12" I-beams 7 x 15 x [illegible] = [illegible]

Rails [illegible] x 30 x [illegible] = 900

5[illegible]

Add 15% for details = [illegible]

Total steel = [illegible]

Floor = [illegible]

[illegible]

Assume weight of floor beam = 1400

Total Dead Load = [illegible] "

[illegible] = [illegible] = [illegible] = [illegible]

Live Load [illegible].

There is a chance for two kinds of loading:-

(1) Street car and uniform load. Use this for moments. (Fig.[illegible])

(2) [illegible] Engine and uniform load. Use this for shear. (Fig.[illegible])

[illegible] + [illegible] x [illegible] = [illegible] + [illegible] = [illegible] = [illegible]

moment of street car.

Middle 12' width = 10 x 100 x 5/15 = 334 [illegible] per

foot floor beam.

Condition of live load for maximum moment :-

Reaction = 2 x 1700 = [illegible]

[illegible] x .34 = 1[illegible]4

Car load = $\underline{12[illegible]0}$

[illegible]

Moment at the center = 1[illegible] x $\frac{17.5[illegible]}{2}$ - 3[illegible]00 x 7

- 1944 x 3 - 12000 x 2 = 14[illegible] - [illegible] - 5832

- 25000 = 14[illegible] - [illegible] = [illegible]470 '" = 11[illegible] "#.

Total moment = [illegible] + [illegible] = [illegible] "# = [illegible]

inch kips,

Section mod. = $\frac{2[illegible]}{12}$ = [illegible]

Use 24" @ 80# I-beam [illegible] = [illegible] [illegible] for [illegible]

For shear set the traction engine against the curb. Traction engine is assumed to cover [illegible] x [illegible]'. Uniform load as before. (Fig. [illegible])

Uniform load at the end of traction engine:-

$\frac{[illegible] \times 100 [illegible]}{15}$ + $\frac{[illegible] 100 \times [illegible]}{15}$ = [illegible] "

Left reaction = $\frac{[illegible] \times 3 \times 17.[illegible]}{1[illegible]}$ + $\frac{[illegible]}{[illegible]}$

+ $\frac{12000 \times 17.[illegible]}{[illegible]}$ = [illegible] + [illegible] + [illegible] = [illegible] "

100 # per sq. ft. alone gives $\frac{[illegible] \times 15 \times 100}{2}$

= 1[illegible]

The other [illegible] = [illegible]

Area of section = 27.[illegible] sq.in. [illegible] is more than enough for [illegible].

Maximum Stringer Reaction.

Roadway Stringers:- (Fig.7)

R = 8000 + 5/15 x 4000 = 9333

$\frac{3 \times 2 \times 100 \times 1.5}{15} = 60$

D.L. = 1[illegible] x 1[illegible] = 210[illegible]

Total = 9[illegible]3 + 60 + 2100 = 11[illegible]9[illegible]

Track Stringers.

R = 8000 + 5000 =	13[illegible]00	
Dead Load =	20[illegible].5	
No uniform live load.	13200.5	Total

Connection Angles on Stringers.

$\frac{3}{4}$" rivets

Art. 53. 12000 x .[illegible] = 5300 # shear.

20000 x .90 = 10000 " bearing.

Single shear = 7570

Double shear = [illegible]70

Bearing Values of Rivets.

$\frac{1}{4}$	5/16	3/8	7/16	$\frac{1}{2}$	9/16
3000	3750	4500	[illegible]	[illegible]	[illegible]

Fo[illegible] Stringers.

Lines AA and FF in bearing and double shear.

$\frac{11493}{750}$ = [illegible] rivets req'd. (Fig.[illegible])

Rivets in floor beam are in single shear or bearing on web of floor beam.

$\frac{11493}{3530}$ = 4 shop rivets or 6 field rivets.

Track Stringers.

Web = 3/8"

$\frac{12806.5}{4500}$ = 3 rivets

Web of floor beam = $\frac{12806.5}{3530}$ = [illegible] rivets or 6 fields.

Use 10" I-beam.

Use 2 angles 4" x [illegible]" x [illegible]/16" x [illegible]'-5½" Standard.

Floor Beam Connection To Post.

In web of floor beam the rivets are in double shear or bearing.

$\frac{15850}{6000}$ = 3 rivets

In post assume single shear, since post should not be thinner than 5/16".

$\frac{15850}{3530}$ = 5 shop rivets or 6 field rivets.

Use 2 angles 6" x 4" x ½" x 1'-5½" long. [illegible]

Loads for Trusses.

1130 # per foot of car track.

75 # per foot of remaining surface.

Per panel per truss L.L.

$1130 \times 7\frac{1}{2} = 8480$

$75 \times 7\frac{1}{2} \times (17.5 - 10) = 4110$

Sum $= 8480 + 4110 = 12590$ # L.L.

Per panel per truss, D.L.

Floor $= 17.5 \times 7\frac{1}{2} \times 105 = 13800$

Rails $= 50 \times 15 = 750$

Total $= 14550$

Steel in floor $= \frac{15}{2} \times 80 = 600$ #

Roadway stringer $= 3\frac{1}{2} \times 35 \times 15 = 1841$

Track stringer $= 1 \times 31.5 \times 15 = 473$ #

Sum $= 600 + 1841 + 472 = 2913$ #

10 lineal feet 6" x 4" x 7/8" angles @ 16#

$= 160$ #

$160 + 2913 = 3073$ say 3070 #.

This makes 1170 # per lineal foot of bridge per truss. Railing = 70 # per lineal foot or 470 # per panel per truss.

Steel from Du Bois's Formula

$\frac{15}{2}$(350 + 2. [illegible] x [illegible]) = [illegible] per panel per truss.

3363 + 135 [illegible] = 1700 lbs per panel for [illegible]

17100 #

[illegible].

$\text{Tan } \theta = \frac{8}{7.5} = 1.0666 \quad \theta = 46^\circ - 50' \quad \sec \theta = 1.46173$

P.L. = W = 17100 #

Chord eg. C. of M. at F.

$\frac{3W \times 3l}{n} - W(l_2 + 2l_2) = \frac{3Wp}{n}$ [illegible]

$= \frac{3 \times 17100 \times 15}{8} = 96000$ #

Chord ff. C. of M. at e.

$\frac{3W \times 3l}{n} - Wl = \frac{3Wp}{n} = \frac{3 \times 17100 \times 15}{8} = 96000$

Chord de. C. of M. at D.

[illegible] $= \frac{2.5}{n}$ [illegible]

$= \frac{2.5 \times 17100 \times 15}{8} = 80200$

Chord ff. C. of M. at C.

$\frac{2Wp}{n} = \frac{2 \times 17100 \times 15}{8} = 64100$ #

Chord Ac. C. of M. at B.

$\frac{2W \times 2l}{n} = \frac{Wp}{n} = \frac{17100 \times 15}{8} = 32100$ #

Diagonals.

Shear in panel FG = 0.

Therefore there is no dead load stress in the diagonals.

Shear in panel CE = $2W - W = W = 17100$

Stress = $17100 \sec\theta = 25000$ " $= Ce = -Dc$

Shear in panel AC = $2W = 34200$

Stress = $34200 \sec\theta = 50000$ " $= Bc = -Ab$.

Posts.

Post Cc.

$Dc \sin\theta + Cc + Bc \sin\theta = 0$

$-25000 \sin\theta + Cc + 50000 \sin\theta = 0$

$Cc = -(50000-25000) \sin\theta = -18000$ "

Post Ee.

$De \sin\theta + Ee + Fe \sin\theta = 0$

$0 + Ee = -25000 \sin\theta = -18000$ "

Live Load Stresses.

$\text{Ratio} = \frac{12660}{17100} = 0.74$

$eg = DF = 71000$

$ce = 59400$

$EF = 47400$

$Ac = 33800$

$Fc = \frac{(1 + 2 + 3 + 4)w'}{5} \sec \theta = 10/5 \times 13140 \times 1.40173$

$= 36810$ #

$AD = -Fc = -36810$

$De = \frac{(1 + 2 + 3)w'}{5} \sec \theta = 6/5 \times 13140 \times 1.40173$

$= 22100$

$Dc = -De = -22100$

$Fg = -Fe = \frac{(1 + 2)w'}{5} \sec \theta = 3/5 \times 13140 \times 1.40173$

$= 11050$ #

Cc.

$Dc \sin \theta + Cc + Fc \sin \theta = 0$

$-22100 \sin \theta + Cc + 36810 \sin \theta = 0$

$Cc = -(36810 - 22100) \sin \theta = -14710 \times .7$[illegible]

$= -10720$

Ee.

$Fe \sin \theta + Ee + De \sin \theta = 0$

$-11050 \sin \theta + Ee + 22100 \sin \theta = 0$

$Ee = -(22100 - 11050) \sin \theta = -11050 \sin \theta = -8060$ #

Design of Upper Chord.

BF.

D.L.		36000
L.L.		71000
Impact L.L.	$\frac{300}{[illegible] + 45}$	58600
		166?00

$C = 16000 - 70\ l/r$

Assume section as follows:-

1 Cover Plate 12" x $\frac{1}{2}$" = 6.00

2 angles 6" x 4" x 5/8" = 11.7[illegible]

17.7[illegible] sq. in.

To find the center of gravity of the section take moments about the top.

$$y = \frac{6 \times .25 + 11.7[illegible] \times 2.53}{17.7[illegible]} = \frac{1.5 + 29.61}{17.7[illegible]} = 1.75"$$

Moments of inertia about the center of gravity.

$$I_a = 1/12 \times 12 \times (.5)^3 + 6.00 \times (1.5)^2 = 13.6[illegible]$$

$$I_b = 2 \times 21.1 + 11.7[illegible] \times (.78)^2 = \underline{49.3[illegible]}$$

61.[illegible]1

$$r = \sqrt{I/A} = \sqrt{\frac{61.[illegible]1}{17.[illegible]}} = 1.87$$

$$f = 18000 - \frac{[illegible] \times 7.5 \times 12}{1.87} = 18000 - 74[illegible] = 14[illegible]$$

Take f = 14000 in designing.

$$\frac{[illegible]}{14000} = 15.9 \text{ sq. in.}$$

$$\text{Rivets req'd.} = \frac{17.7[illegible] \times 14000}{9000} = \text{[illegible] shop or 47 field}$$

	[illegible]
D.L.	64100
L.L.	47[illegible]00
Impact	[illegible]
	148800 Total

Use the same [illegible] for IF.

Design of End Post.

AT.

Use the same section as for IF [illegible].

Length $= \sqrt{[illegible]^2 + 7.[illegible]^2} = \sqrt{1[illegible]0.[illegible]} = 10.[illegible]$

D.L.	30000
L.L.	7[illegible]6[illegible]0
Impact	[illegible]4[illegible]00
	11[illegible]8[illegible]10

$f = 16000 - 70\ l/r = 16000 - \frac{[illegible]}{1.[illegible]}$

$= 12300$

Area req'd $= \frac{118[illegible]10}{12700} = 9.[illegible]$ sq. in. [illegible]

31 rivets req'd [illegible] in upper chord.

Design of Lower Chord Tension Members.

Lower Chord [illegible].

D.L.	[illegible]0000
L.L.	[illegible]2000
Impact $\frac{[illegible]}{300 + [illegible]}$	56[illegible]00
	[illegible]3[illegible]300

Net area req'd $= \frac{[illegible]}{1[illegible]0} = 1[illegible].0$ sq. in.

Try 2 angles [illegible]" x [illegible]" x 7/8" [illegible] $= 1[illegible].[illegible]$

Deduct 2 rivet holes @ [illegible] = $\frac{1.[illegible]}{1[illegible].[illegible]}$

Rivets req'd in [illegible] chord:

$\frac{11.42 \times 10000}{8000}$ = 28 shop rivets or 41 field.

Lower Chord[illegible].

D.L. 60000

L.L. 58400

Impact $\frac{300}{300 + 75}$ $\frac{42000}{157800}$

Net area req'd = $\frac{187800}{16000}$ = 11.7 sq. in.

Try 2 angles 6" x 4" x 1/2" = 1[illegible] sq. in.

Deduct 2 rivet holes x .88 = 1.50

12.[illegible] sq. in.

$\frac{12.56 \times 10000}{8000}$ = [illegible] shop rivets or 76 field.

Design of Post.

[illegible].

D.L. 18652

L.L. 10[illegible]20

Impact I.?. $\frac{300}{300 + 60}$ $\frac{9000}{37870}$ Compression.

f = 16000 - 50 l/r

Try 2 angles 4" x 3" x 5/16" Area = 4.18 sq. in.

I = 2 x 3.4 = 6.8

$r = \sqrt{I/a} = \sqrt{\frac{6.8}{4.18}} = \sqrt{1.63} = 1.27$

$f = 16000 - \frac{50 \times 8 \times 12}{1.27} = 10000$

Area req'd = $\frac{[illegible]}{10000}$ = .[illegible] sq. in. [illegible]

Rivets req'd = $\frac{1.1[illegible] \quad 10[illegible]00}{8000}$ = 6

Design of Diagonals.

[illegible] Tension Member.

D.L. [illegible]

L.L. [illegible]810

Impact L.L. $\frac{[illegible]}{300 + [illegible]0}$ [illegible]

[illegible] 1[illegible]

Net area req'd = $\frac{11[illegible]41[illegible]}{16000}$ = 7.[illegible] sq. in.

Try 2 angles 4" × 3" × [illegible]" Gross area = [illegible]

Deduct 2 rivet holes @ .[illegible]0 = 1.[illegible]

Net area = 7.[illegible]

Rivets req'd = $\frac{7.[illegible] \times 1[illegible]}{8000}$ = 1[illegible] rivets

[illegible] Tension Member.

D.L. [illegible]

L.L. [illegible]

Impact L.L. $\frac{[illegible]}{300 + [illegible]0}$ 1[illegible]

[illegible]

Net area req'd = $\frac{[illegible]}{16000}$ = [illegible] sq. in.

Try 2 angles 4" × 3" × ½" = 6.[illegible] sq. in.

Deduct 2 rivet holes @ .[illegible] = [illegible]

[illegible] [illegible]

Diagonals – Compression

Diagonal [illegible] - Compression.

Dc = -Le = 65550

f = 16000 - 70 l/r

Try 2 angles [illegible]" x $3\frac{1}{2}$" x $\frac{3}{4}$"   A = 10.12 sq.in.

I = 2 x [illegible] = [illegible]

$r = \sqrt{I/A} = \sqrt{\frac{[illegible]}{10.12}} = 1.05$

$f = 16000 - \frac{70 \times 11 \times 12}{1.05} = 7200$

$\frac{65550}{7200} = 9.10$ sq. in.   [illegible]

Rivets req'd $= \frac{10.12 \times [illegible]}{5000} =$ [illegible] or [illegible]

Design of Diagonals in End Panel.

Dc (compression) and Dt (tension) = [illegible].

Live Load Stresses Only.

Dc.

I.L.   11050

Impact L.F. $\frac{300}{300 + 30}$ $\frac{[illegible]}{[illegible]}$

f = 16000 - 70 l/r

Try 2 angles 3" x 3" x $\frac{1}{2}$"   Area = 5.[illegible] sq.in.

I = 2 x 2.2 = 4.4

$r = \sqrt{I/A} = \sqrt{\frac{4.4}{5.5}} = .895$

$f = 16000 - \frac{70 \times 11 \times 12}{.895} =$ [illegible]

[illegible]

$\frac{11250}{1000}$ = 5.1[illegible] sq. in. req'd [illegible]

Rivets req'd = $\frac{[illegible] \times 4000}{8000}$ = 3 rivets

Lateral Bracing.

(Fig. 9) According to specifications the wind load is 300 pounds per lineal foot dead load and 150 pounds per lineal foot live load. The panel dead load is 15 x 300 = 4500 #.

Tan $\theta = \frac{1[illegible].5}{15}$ = 1.[illegible]

θ = 50° - 30'

sec θ = 1.[illegible]

L.L. per panel = 15 x 150 = 2250

Dead Load Stresses.

Left reaction = $2\frac{1}{2}$ x 4500 = 11250

Shear in panel ED for D.L. = 0

Therefore no dead load stresses in diagonals.

Shear in panel CD.

Shear = $(2\frac{1}{2} - \frac{1}{2} - 1)w = w$ = 4500

Stress in diagonals = 4500 sec θ = 7100

Shear in panel B[illegible].

Shear = $(2\frac{1}{2} - \frac{1}{2})w = 2w$ = 9000

Stress in diagonals = 9000 sec θ = 14200

Live Load Stresses.

Panel FG. Live Load up to f.

Shear $= \frac{(1 + 2)W}{5} = 3/5W = 3/5 \times 2250 = 1350$

Stress in diagonals $= 1350 \sec\theta = 2140$ "

Panel GH. Live load up to e.

Shear $= \frac{(1 + 2 + 3)W}{5} = 6/5 \times 2250 = 2700$

Stress in diagonals $= 2700 \sec\theta = 4275$

Panel AG. Live load up to d.

Shear $= \frac{(1 + 2 + 3 + 4)W}{5} = 2W = 2 \times 2250 = 4500$

Stress in diagonals $= 4500 \sec\theta = 7130$

Total stress in Fg $= 2140$

Area req'd $= \frac{2140}{16000} = .134$

Try one angle $3\frac{1}{2}" \times 3" \times 3/8"$ Area $= 2.3$ sq.in.

Net area $= 2.3 - .38 = 1.92$ sq.in.

Shop rivets req'd $= \frac{1.92 \times 16}{8} = 4$ or 6 field.

D.L. 4215

L.L. 7130

11345

Area req'd $\frac{11345}{16000} = .71$ sq.in.

Try 1 angle $3\frac{1}{2}" \times 3" \times 3/8"$ Area $= 2.3$ sq.in.

Net area = 2.70 - .78 = [illegible] sq.in.

4 shop rivets or 2 field.

AC

D.L. 12840

L.L. [illegible]

21?00 Total

Area req'd $\frac{21?00}{16000}$ = 1.3? sq. in

Try 1 angle 3½" x 3" x 7/8" Area = 2.7? sq.in.

Net area = 2.70 - .78 = 1.92 sq.in.

4 shop rivets or 2 field.

Design of End Span.

The dimensions of this bridge are the same as for the middle span except that the length is sixty feet instead of seventy five feet. There will be four panels of fifteen feet and the height will be eight feet as was the case in the preceding design.

Design of Floor.

The design of the floor will be the same in this case as in the preceding design.

Design of Stringers.

Since the panel length is the same for both bridges, the design of the stringers will be the same.

Design of Floorbeam.

The floorbeams will be the same for both bridges. The connection angles of the stringers and floorbeams will also be the same.

Loads for Trusses.

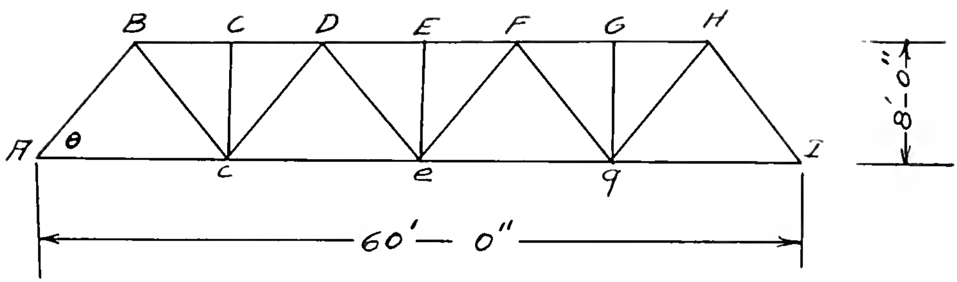
The weight of the floor per panel will be the same or 13800 #

Steel from Du Four's formula:-

$\frac{15}{2}(250 + 7.5 \times 60) = 5[illegible]00$ "

13800 + 3000 = 16800 # per panel per truss D.L.
The live load per panel per truss will be the same in both cases = 12590 #

Trusses.



θ = 46° - 51' sec θ = 1.46[illegible]3

D.L. = w = 16800 #

Chord EF. c. of m. at e.

$$\frac{1\frac{1}{2}w \times 2p - wp}{h} = \frac{2wp}{h} = \frac{2 \times 16800 \times 15}{8} = 63000$$

Chord ce. c. of m. at D.

$$\frac{1\frac{1}{2}w \times 1\frac{1}{2}p - w \times \frac{1}{2}p}{h} = \frac{2\frac{1}{4}wp - \frac{1}{2}wp}{h} = \frac{1\frac{3}{4}wp}{h}$$

$$= \frac{1.75 \times 16800 \times 15}{8} = 55125$$

Chord BD. c. of m. at c.

$$\frac{1\frac{1}{2}w \times p}{h} = \frac{1\frac{1}{2}wp}{h} = \frac{1.5 \times 16800 \times 15}{8} = 47250 \#$$

Chord Ac. c. of m. at B.

$$\frac{1.5w \times .5p}{h} = \frac{.75wp}{h} = \frac{.75 \times 16800 \times 15}{8} = 23625 \#$$

Diagonals.

Shear in panel BC = $1\frac{1}{2}w - w - w = -\frac{1}{2}w = -8400$

Stress = 8400 sec θ = 13000 = [illegible] = $-T_b$

Shear in panel CD = $1\frac{1}{2}w - w = \frac{1}{2}w = 8400$ "

Stress = 8400 sec θ = 13000 " = [illegible] = -Dc

Shear in panel AC = $1\frac{1}{2}w$ = 25200 "

Stress = 25200 sec θ = 40000 = [illegible] = -Ab.

Posts.

Post Cc.

Dc sin θ + Cc + Bc sin θ = 0

-13000 sin θ + Cc + 40000 sin θ = 0

Cc = -(40000 - 13000) sin θ = -27000 sin θ = -17350

Post Ee.

Fe sin θ + Ee + De sin θ = 0

13000 sin θ + Ee + 13000 sin θ = 0

Ee = -2 × 13000 sin θ = -17000 "

Live Load Stresses.

Ratio = $\frac{12600}{16800}$ = .75

DE = 17200

ce = 11250

BD = 35400

Ac = 17700

$Bc = \frac{(1 + 2 + 3)W'}{4} \sec \theta = 6/4 \times 1[illegible] \times 1.40[illegible]$

$= 27600$

$AB = -Bc = -27600$

$Ce = \frac{(1 + 2)W'}{4} \sec \theta = 3/4 \times [illegible] \times 1.40177 = 13800$

$Dc = -De = -13800$

$Dc \sin \theta + Cc + Bc \sin \theta = 0$

$-13800 \sin \theta + Cc + 27600 \sin \theta = 0$

$Cc = -(27600 - 13800) \sin \theta = 10000$ #

Design of Upper Chord.

D.L.	63000
L.L.	47[illegible]00
Impact I.L. $\frac{300}{300 + 30}$	42000
	153100

$f = 15000 - 70\ l/r$

Assume section as follows:-

1 Cover plate	12" x 3/8" =	4.5 sq. in.
2 angles 6" x 4" x 1/2" =		9.50
		14.00 sq.in.

To find the center of gravity of the section take moments about the top.

$$y = \frac{4.5 \times .187 + 9.5 \times 2.365}{14.00} = \frac{.84 + 22.45}{14.00} = 1.675$$

Moments of inertia about the center of gravity:-

$I_a = 1/12 \times 18 \times (.375)^3 + 4.5 \times (1.4[illegible])^2 = 1[illegible].0[illegible]$

$I_g = 2 \times 1[illegible] + 9.[illegible]0 \times (.31[illegible])^2 =$ $\underline{[illegible]}$

[illegible]

$r = \sqrt{I/a} = \sqrt{\frac{[illegible]}{11.0[illegible]}} = \sqrt{[illegible].3[illegible]} = 1.0[illegible]$

$f = 18000 - \frac{70 \times 7.5 \times 12}{1.0[illegible]}$

$= 18000 - [illegible] = 1[illegible]$

$\frac{153100}{13880} = 11.0[illegible]$ sq. in. req'd [illegible]

Rivets req'd $= \frac{14.[illegible] \times 13880}{8000} = [illegible]$ shop or [illegible] field.

EL.

D.L.	47[illegible]50
L.L.	[illegible]400
Impact	30800
	[illegible]450 Total

Use same section as for [illegible].

Design of End Post AT.

Length = 12' - 0"

Use same section as for EI.

D.L.	30[illegible]
L.L.	37[illegible]
Impact	[illegible]000
	[illegible]7400

$c = 1800 - 60\ l/r = 1[illegible] - \frac{60 \times 11 \times [illegible]}{[illegible]}$

$= 18000 - [illegible] = 1[illegible]100$

Area req'd $= \frac{[illegible]}{[illegible]1100} = [illegible]$ [illegible] [illegible]

2[illegible] shop rivets 39 field.

Design of Lower Chord Tension Members.

[illegible]

D.L.	[illegible]5100
L.L.	[illegible]1[illegible]0
Impact $\frac{[illegible]}{[illegible] + [illegible]}$	33000
	1[illegible]

Net area req'd $= \frac{1[illegible]}{18000} = [illegible]$

Try 2 angles 6" x 4" x ½" = [illegible]

Deduct 2 rivet holes [illegible] = [illegible]

[illegible] sq. in.

$\frac{1.[illegible] \times [illegible]}{6000} = 1[illegible]$ [illegible] rivets or 27 fi[illegible] rive[illegible]

Use the same section for the whole lower chord.

Design of Post.

[illegible]

D.L.	[illegible]050	
L.L.	1[illegible]	
Impact L.L. $\frac{300}{300 + 45}$	[illegible]	
	[illegible]	Comp.

$c = 18000 - 60\ l/r$

Try 2 angles 4" x 3" x 5/16" Area = [illegible] sq. in.

$I = 2 \times [illegible].4 = [illegible]$

$r = \sqrt{I/a} = \sqrt{\frac{[illegible]}{[illegible]}} = 1.[illegible]$

$f = 16000 - \frac{70 \times [illegible] \times 12}{1.[illegible]} = 1[illegible]$

Area req'd $= \frac{[illegible]}{1[illegible]} = 3.6[illegible]$ sq. in. sect. [illegible].

Design of Diagonals.

Fe—Tension member.

D.L.	36[illegible]00
L.L.	27000
Impact L.L. $\frac{300}{300 + 45}$	[illegible]
	[illegible]

Net area req'd $= \frac{[illegible]}{16000} = [illegible].[illegible]5$ sq.in.

Try 2 angles 4" x 3" x 5/16" Gross area = [illegible]

Deduct 2 rivet holes x .4[illegible] = [illegible]

[illegible]

Fe—Compression member.

D.L.	13[illegible]0
L.L.	13[illegible]0[illegible]
Impact $\frac{300}{300 + 45}$	[illegible]
	[illegible]

$f = 16000 - 70\ l/r$

Try 2 angles 4" x 3" x [illegible]" A = [illegible]

$I = 2 \times [illegible] = 10.[illegible]$

$r = \sqrt{I/a} = \sqrt{\frac{10}{[illegible]}} = \sqrt{1.[illegible]} = 1.[illegible]$

$$f = 16000 - \frac{80 \times 11 \times 12}{1.34} = 16000 - 8520 = 7480$$

$\frac{38060}{7480} = 5.08$ sq.in. req'd. O.K.

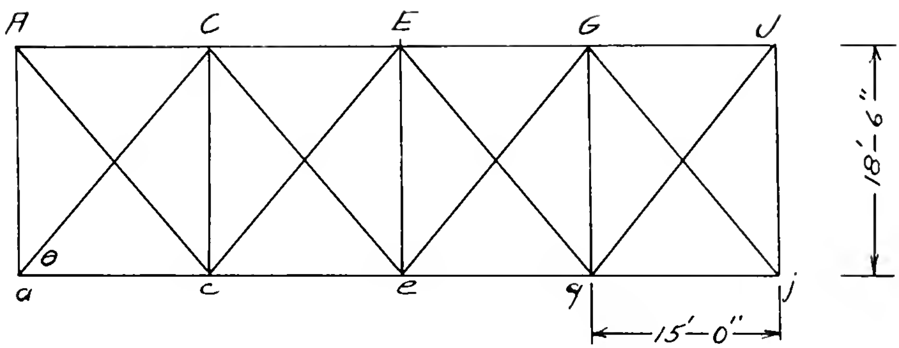
Rivets req'd $= \frac{2.5 \times 7480}{8000} = 7$ shop or 10 field

Lateral Bracing.

D.L. per panel = 4500 same as in previous design.

L.L. per panel = 2250 " " " " " .

Dead Load Stresses.



sec θ = 1.58733

Shear in panel CE = (2 - 1½ - 1)w = ½w = 2250

Stress in diagonals = 2250 sec θ = 3570

Shear in panel AC (2 - ½)w = 1½w = 6750

Stress in diagonals = 6750 sec θ = 10700

Live Load Stresses.

Panel CE. Live load up to e

Stress in diagonals = 1[illegible] sec θ = [illegible]

Panel AC. [illegible]

Tan = [illegible] = [illegible]

Stress in diagonals = [illegible] sec = [illegible]

Stress in Ac.

D.L. [illegible]

L.L. [illegible]

[illegible]

Area req'd = $\frac{11[illegible]}{16000}$ = .[illegible]

Try 1 angle [illegible]" x [illegible]" x 7/[illegible]" Area = [illegible] sq. in.

Net area = [illegible] − .[illegible] = 1.3[illegible] sq. in.

Shop rivets = $\frac{1.[illegible] \times [illegible]}{[illegible]}$ = [illegible] or [illegible] rivets.

Design of Reinforced Concrete Abutment.

Design footings and piling for highway abutment, footings to take all bearing. Then footings to be set on piling.

Piling:- Safe load per pile = $\frac{2wh}{s+1}$

w = weight of hammer = [illegible]"

h = drop of hammer = [illegible]' - [illegible]".

s = penetration of pile for last blow = 1".

Weights and loading:-

Concrete = 150 # per cubic foot.

Weight of bridge determined from design of a 75' Highway Bridge - Class C.

Weight of Live Load - Coopers Specifications.

Bearing Abutment:-

Coarse gravel.

[illegible] tons per square foot allowable.

Load.

Weight of bridge = [illegible] #.

$\frac{[illegible]}{2}$ = [illegible] # carried by each abutment.

= [illegible] # carried by each bridge seat.

Trusses = [illegible]' [illegible]" center to center.

Truss span = [illegible]' - [illegible]".

Place traction engine with heavier wheel load directly over the abutment and at one end of the bridge. The end of the bridge receives 7/4 of the weight of the heavier wheel load plus 3/4 of the weight of the lighter wheel load.

Heavier wheel = 8 tons = 16000 #

Lighter wheel = 4 tons = 8000 #

7/4(16000 + 50/60 x 8000) = 17000 # for each bridge seat. Distribute load over 12' - 0".

$\frac{17000}{12}$ = 1417 # per lineal foot L.L.

$\frac{51350}{12}$ = 4271 # per lineal foot D.L.

L.L. of 100 # per square foot of remaining floor space gives $\frac{[illegible] \times 100 \times 1.5}{12} + \frac{15 \times 100 \times [illegible]}{12}$

= 30 + 750 = 780 # per lineal foot.

D.L. = 4271 # per lineal foot.

L.L. = 1417 + 780 = 2197 # per lineal foot.

Total = 4271 + 2197 = 6478 #

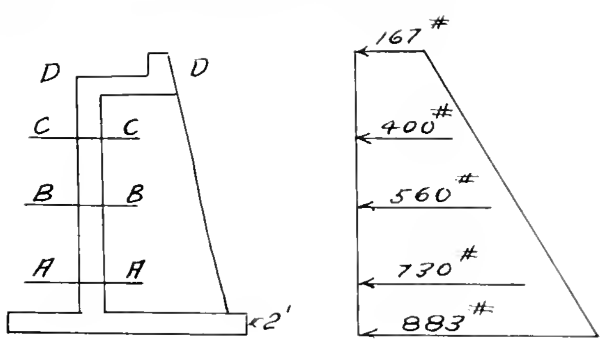


Figure [illegible].

Surcharge = [illegible] = [illegible]

Counterfort:-

Pressure at top = [illegible] = [illegible]

Pressure at base = [illegible] + [illegible] = [illegible] = [illegible]

Resultant = [illegible] + [illegible] × [illegible] = 11877.5 #

Design of section A – A.

Unit pressure at A – A = 750 # per sq. ft. [illegible]

Design as a simple beam [illegible] span.

$M = 1/12\, wl^2 = 1/12 \times 750 \times$ [illegible] $\times 12 =$ [illegible] "#

$d = \sqrt{\frac{M}{Kb}} = \sqrt{\frac{[illegible]}{[illegible]}} = \sqrt{35.[illegible]} =$ [illegible]

Use a uniform thickness of 12".

$f_s = 15000$ $\quad j = 7/8$ $\quad k = 3/8$ $\quad$ [illegible]

For shear: [illegible] = [illegible] = [illegible] #

$v = \frac{V}{bjd}$

$d = \frac{V}{bv} = \frac{[illegible]}{12 \times [illegible]} =$ [illegible]" [illegible]

Steel area, $A = \frac{M}{f_s j d} = \frac{[illegible]}{15000 \times 7/8 \times [illegible]} =$.[illegible] sq. in.

Use ½" [illegible] rods spaced [illegible]" centers, $A =$.[illegible] sq. in.

Section [illegible] - [illegible].

[illegible]' - 0" above the base.

Unit pressure = [illegible] per sq.in. (scaled)

[illegible] = 1/12 x [illegible] = [illegible]40 ".

Steel area = A = $\frac{[illegible]}{s_s j d}$ = $\frac{[illegible]40}{[illegible] \times 7/8 \times [illegible]}$ = [illegible] sq.in.

Use $\frac{1}{2}$" ∅ rods spaced [illegible]" centers. Area = .[illegible] sq.in.

Section [illegible] - [illegible].

[illegible]' - 0" above the base.

Unit pressure = [illegible] per sq.in. (scaled)

[illegible] = $1/12 wl^2$ = 1/12 x [illegible] x 24 x 12 = [illegible] "

Steel Area = A = $\frac{[illegible]}{s_s j d}$ = $\frac{[illegible]}{[illegible] \times 7/8 \times [illegible]}$ = .[illegible] sq.in.

Use $\frac{1}{2}$" ∅ rods spaced 12" centers. Area = .[illegible] sq.in.

Section [illegible] - [illegible].

Area Steel req'd = .[illegible] sq.in. (proportionally)

Use $\frac{1}{2}$" ∅ rods spaced [illegible]" Area = .[illegible] sq.in.

Steel to tie Slab to Counterfort.

Section A - A.

[illegible]' - 0" above the base.

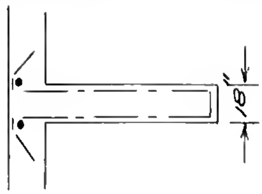
Unit pressure = [illegible] " per sq.in.

Reaction of slab on counterfort = [illegible] x [illegible] = [illegible]

Steel req'd to tie slab to counterfort equals

$\frac{[illegible]40}{[illegible]000}$ = .3[illegible] sq.in.

Bend rods U-shaped as shown in the figure.



$\frac{.[illegible]}{2}$ = .1[illegible] sq.in. area of each rod.

Use 7/8" ∅ rods spaced [illegible]" centers.

In order to develop the full strength of the rod, the length must be 32.5 diameters. The length required = [illegible] x 7/8 = [illegible]" long. This is for bond stress of [illegible] #. Since the rods are longer than [illegible] inches. Length of counterfort at this point = [illegible]' - 6". Extend rods to hook on rods at back of counterfort. Use the same size rods up the entire wall.

Section [illegible] - [illegible].

The pressure = 5[illegible]0 #

Reaction of slab at counterfort = [illegible] x [illegible]" = 44[illegible]

Steel req'd to tie slab to counterfort = $\frac{4480}{16000}$

= .280 sq.in.

Bend rods to be U-shaped.

$\frac{.280}{2}$ = .140 sq.in. req'd

Use 3/8" ∅ rods spaced [illegible]" centers.

Section C – C.

Unit pressure = 400 #

Reaction of rib = [illegible] x 400 = 3200 #

$\frac{3200}{15000}$ = .214 sq.in steel req'd

Bend four rods to [illegible].

$\frac{.214}{2}$ = .107 sq.in. per rod.

Use 5/8" ø rods [illegible] 15" [illegible] centers.

Vertical [illegible] in Counterfort.

Total weight of earth above base :-

= (3½ x 17.7 x 100 + [illegible] x [illegible] 100) =

(2775 + 15375) = 19[illegible] #

Total [illegible] req'd = $\frac{[illegible]}{15000}$ = 1.28 sq.in. to hold up rib.

Use 13 - ¾" ø rods [illegible] 1.64 = 10.[illegible]

Area of [illegible] in back of counterfort.

At the base:-

M = $\frac{167 + 630}{2}$ x [illegible] x 12 [illegible] = [illegible]0000

M = f_sAjd d = 3.75

A = $\frac{8420000}{15000 \times .87 \times [illegible] \times 12}$ = [illegible] = [illegible] sq.in.

Counterfort = 15" [illegible] = 1.51 sq.in for one foot.

Take section [illegible] – [illegible]" from the [illegible].

$M = \frac{580 + 167}{2}$ [illegible] 12.5 [illegible] x 12 x [illegible] = [illegible]

$d = 7.75$

$A = \frac{M}{f_s jd} = \frac{[illegible]}{18000 \times [illegible] \times 7.75 \times [illegible]} = [illegible] = [illegible]$ sq. in.

Use 1" ∅ rods [illegible] " centers $A =$ [illegible].00 sq. in.

Run these rods all the way down to the base.

Section [illegible] the [illegible]:-

Additional steel [illegible]:-

[illegible] – [illegible] = [illegible] sq. in.

Use [illegible] 1/2" ∅ [illegible] [illegible]" centers.

Resultant Pressure on Base.

Weight of Counterfort:-

$(3\frac{1}{2} \times 18.5 + [illegible] \times 8.75)1.5 \times 150 = [illegible]$

Average weight over [illegible]' of wall $= \frac{[illegible]}{5} = [illegible]$

Resultant [illegible] [illegible] ' from [illegible] of [illegible].

Weight of wall and point of application [illegible].

$$\frac{[illegible]5 \times 1.5 \times 150)[illegible] + ([illegible] \times 150)[illegible] + (1.5 \times 20.5 \times 150)[illegible] + ([illegible] \times 150)[illegible]}{\text{Total Weight}} =$$

$$= \frac{[illegible]570 + [illegible] + [illegible] + [illegible]}{675 + [illegible] + [illegible] + [illegible]} = \frac{[illegible]}{11070} = [illegible]$$

The point of application of the weight of the abutment is [illegible] feet from the [illegible] of the wall.

Total Vertical Force and Point of Application.

$$\frac{(\ldots \times 12 \ldots 150)\ldots + (\ldots \times 150)\ldots + (11073 \times 10.55) + (5000 \times 7.02) + \ldots + \ldots}{\text{Total Vertical Weight}}$$

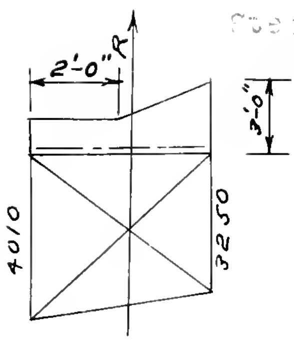
$$= \frac{501780}{57945} = [illegible] \text{ feet.}$$

Horizontal force = [illegible] acts 3.6 feet above the base. The resultant falls within the middle third. Therefore ....

Eccentricity [illegible] feet.

$$\text{Pressure Toe} = \frac{57945}{17}\left(1 + \frac{6 \times \ldots}{17}\right) = [illegible] \text{ per sq. ft.}$$

$$\text{Pressure Heel} = \frac{57945}{17}\left(1 - \frac{6 \times \ldots}{17}\right) = [illegible] \text{ per sq. ft.}$$



Steel at Top of Wall.

$$\text{Shear} = \frac{4010 + 3250}{2} \times [illegible]$$

$$= 18[illegible]\ \#$$

Depth req'd for shear equals

$$\frac{18\ldots}{1\ldots} = [illegible] \text{ inches}$$

Make section 3' - 0" and slope to a point [illegible] ft. from the front.

$$\text{B.M.} = \frac{4010 + 3250}{2} \times [illegible] \times [illegible] \times 12 = 450[illegible]\ \text{"\#}$$

$$d = [illegible] - [illegible] = [illegible]\ \text{".}$$

$A = \frac{[illegible]}{f_s j d} = \frac{[illegible]}{15000 \times .87 \times [illegible]} = [illegible]$ sq. in.

Use 3/8 " ϕ rods spaced [illegible]" centers.

Steel in Base.

Consider beam from counterfort to counterfort 12" wide. [illegible] = [illegible] = [illegible] = [illegible]

$M = 1/12 \, wl^2 = 1/12 \times [illegible] \times [illegible] \times [illegible] = [illegible]$

Area steel req'd $= \frac{M}{f_s j d} = \frac{[illegible]}{15000 \times .87 \times [illegible]} = .[illegible]$ sq. in.

Use 7/16 " square rods spaced [illegible]" centers for first 5 feet from [illegible] wall.

For next $3\frac{1}{2}$ ft. to back of vertical wall.-

$w = 1[illegible].5 \times 100 = [illegible]$ #

$M = 1/12 \, wl^2 = 1/12 \times [illegible] \times [illegible] \times 12 = [illegible]$

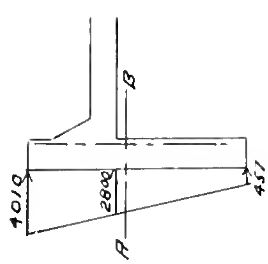
Area steel req'd $= \frac{M}{j d f_s} = \frac{[illegible]}{[illegible] \times .87 \times [illegible]} = .[illegible]$ sq. in.

Use 7.16" square rods spaced [illegible]" centers to back of vertical wall.

Rods in Upper Part of Base.

Section A - B.

Unit pressure due to eccentricity of resultant upward on base = 2800 # per square inch.



Unit pressure due to earth above at section (A - B) = 1[illegible].[illegible] $\times$ 100 = [illegible]

[illegible] = [illegible] − [illegible] = [illegible]

[illegible] = 3/12 [illegible]^2 = [illegible] = [illegible]

Area [illegible] = $\frac{[illegible]}{s_j d}$ = $\frac{[illegible]}{[illegible]}$ = [illegible]

[illegible] 7/16 " [illegible]

[illegible] intermediate [illegible]

Section [illegible]' − [illegible]" [illegible]

Total horizontal [illegible] = [illegible]

[illegible] = [illegible] = [illegible]

A = $\frac{M}{s_j d}$ = $\frac{[illegible]}{[illegible]}$ = [illegible] sq.in.

Use 1" square [illegible]

Area = [illegible]

[illegible]

Unit pressure [illegible] = $\frac{[illegible] \times 100}{[illegible]}$ = [illegible]

Therefore total horizontal pressure [illegible]

$\frac{[illegible]}{[illegible]}$ x [illegible] = [illegible]

[illegible] = [illegible] = [illegible]

A = $\frac{M}{s_j d}$ = $\frac{[illegible]}{[illegible]}$ = [illegible] sq.in. [illegible]

[illegible] − [illegible] = [illegible] sq.in. [illegible]

Use $1\frac{1}{2}$" square [illegible]

to steel [illegible]

Design of Pi[illegible] [illegible] Foundation.

The maximum lo[illegible]d on the p[illegible] [illegible] the traction engine and [illegible] [illegible] [illegible] [illegible] by [illegible] pier.

(16000 + [illegible] x [illegible]) = [illegible] [illegible] engine.

(10000 + [illegible] x [illegible]) = [illegible] [illegible] car.

[illegible] D.L. [illegible] on [illegible] surface = [illegible] "

½ D.L. of 75' – 0" bridge = [illegible]

½ D.L. of [illegible]' – 0" bridge = [illegible] "

[illegible] + [illegible] + [illegible] + [illegible] + [illegible] = [illegible] # is the maximum reaction and load on [illegible] pier.

See blue print for [illegible] [illegible] of the pier.

Weight of concrete = [illegible] #

Weight of [illegible] = <u>[illegible]</u> "

Total wt. of pier = [illegible] "

Total weight to be [illegible] by pile foundation:–

= [illegible] + [illegible] = [illegible]

[illegible] pile [illegible] by $\frac{[illegible]}{+}$

[illegible] = weight of [illegible] = [illegible] #

[illegible] = Drop of [illegible] = [illegible]' – 0"

[illegible] = penetration of last blow = [illegible]"

[illegible]

Estimate of [illegible].

Cubic yards of concrete in part [illegible]:-

$V = \frac{h}{2}(a_1 + a_2)$

$a_1 = ([illegible] \times 30.7) + ([illegible]) = 18[illegible]$ [illegible].

$a_2 = (5.125 \times 70.75) + ([illegible]) = [illegible]$

$V = \frac{25}{2}(188 + [illegible]) = 12.5 \times [illegible]$

= 6160 cubic feet = [illegible] cubic yards.

Concrete in [illegible]:-

= ([illegible] x 12.125 x 15.25) + ([illegible] x 7 x [illegible])

= 1030 + [illegible] = 14[illegible] cu. ft. = [illegible] cu. yds.

Cubic yards of concrete in [illegible] pier:-

[illegible] + 53.[illegible] = [illegible].

Cubic yards of concrete in [illegible]:

[illegible] x 280.2 = [illegible] cubic yards.

The following [illegible] Mettle's Book on Cost Data. The examples follow-ed were similar to the [illegible] and abutment de-signed. The conditions [illegible] results were [illegible] similar to those found in this problem.

[illegible] Report of Pile [illegible] Cost per [illegible].

Cement	[illegible]
Sand	[illegible]
Gravel	.1[illegible]
Foreman @ [illegible]	[illegible]
Labor @ [illegible]	1.00[illegible]
Engine man @ [illegible]	.1[illegible]
Finish coat @ [illegible]	.0[illegible]
Carpenter @ [illegible]	.[illegible]
Forms at $ 2[illegible], [illegible]	.[illegible]
Wire, nails etc.	.0[illegible]
Pro rated plant cost	.[illegible]
Coffer dam	.[illegible]
Coffer dam excavation	.[illegible]
Total	1.[illegible]

[illegible] = [illegible]. [illegible] of [illegible].

Cubic Yards of Concrete in the [illegible] :-

[illegible] = ([illegible] $\frac{[illegible] + [illegible]}{2}$ [illegible]) + ([illegible] $\frac{[illegible]}{2}$ [illegible]) [illegible],

= [illegible] cubic feet = [illegible] cubic yards.

Extra section [illegible] :-

([illegible] x [illegible]) = [illegible] = [illegible] cubic yards.

Vertical wall = ([illegible] x 1[illegible] x 104) = 1[illegible] cubic feet.

= [illegible] cubic yards.

Cast = $\left(1_2 \times \frac{[illegible] + [illegible]}{2} \times 7.[illegible]\right)$ = 77.[illegible] cub.ft. [illegible]

= 2.5 cubic yards.

[illegible]pot = $\left(1_2 \times [illegible] \times \frac{[illegible] + [illegible]}{2}\right)$ = [illegible] = [illegible] cu.yds.

Counterforts:

$2 \times \frac{9 + 5}{2} \times 10.5 \times 1_2$ = [illegible] cu.ft. = [illegible] cu.yds.

$[illegible] \times \frac{7.[illegible] + [illegible]}{2} \times 10.[illegible] \times 1_2$ = [illegible] cu.ft. = [illegible] cu.yds.

$[illegible] \times 6/2 \times 12.5 \times 1_2$ = 7[illegible] cu.ft. = [illegible] cu.yds.

$[illegible] \times 4/2 \times 12 \times 1_2$ = 72 cu.ft. = [illegible] cu.yds.

$[illegible] \times 2.5 \times [illegible] \times 1_2$ = 17.9 cu.ft. = .[illegible] cu.yds.

Total = [illegible] + [illegible] + [illegible] + [illegible] + [illegible] + [illegible] + [illegible]

+ 5.5 + 2.7 + .2 = 20[illegible].1 cubic yards in one abutment.

2 × 20[illegible].1 = 414.2 cubic yards of concrete in the two abutments.

Weight of Steel Reinforcing.

In figuring the weight of the steel reinforcing the lengths of the rods were scaled off the blue print and multiplied by the weight per foot of rod as given in the hand book of the Carnegie Steel Company.

The following [illegible] abutment.

[illegible] - [illegible] [illegible] [illegible] .[illegible] = [illegible] "

" - " " [illegible] [illegible] .[illegible] = 4[illegible] "

" - [illegible] " [illegible] .[illegible] = [illegible] "

" - " " [illegible] .[illegible] = [illegible] "

" - Trim. " [illegible] x .[illegible] = [illegible] "

Counterforts:

Vert. steel [illegible] x [illegible] .[illegible] = [illegible] "

At [illegible] 16 x [illegible] x [illegible] = [illegible] "

" " [illegible] 20 x [illegible] = [illegible] #

Hor. steel 5 x 100 .[illegible] = 15[illegible] "

Wall :

Vert. steel [illegible] x [illegible] .[illegible] = [illegible]

Hor. steel [illegible] x [illegible] .[illegible] = [illegible]

[illegible] and parapet = [illegible] x [illegible] = [illegible]

Total weight of one half of the abutment equals 5[illegible]69.40 # weight of [illegible].

[illegible] x 5[illegible]69.[illegible] = 11[illegible] weight of [illegible] abutment.

[illegible] x 11378.8 = 22673.[illegible] # weight of [illegible] abutments.

Cost of Abutments.

Material	Cost per cu.yd.
Cement @ $ 3.73	$ 4.43
Sand @ $.50½	.53
Gravel @ .50½	.45
Lumber @ $ 35.33	.83
Piles @ $.22	.06
Machinery	.10
Wire and nails	.10
Lubricating oil	.01
Fuel	.13
	$ 6.54

Labor	Cost per cu.yd.
Excavation for foundation	$ .34
Building and removing forms	.57
Driving piles in foundation	.11
Placing steel reinforcement	.16
Mixing concrete	.38
Placing concrete	.17
Pumping water	.03
Cleaning and storing machines	.10
	$ 1.86
	6.54
Total material & labor	$ 8.40

Cost of [illegible] [illegible] [illegible] [illegible]
= 2[illegible]7[illegible]. [illegible] = $ [illegible]
Total [illegible] [illegible] = [illegible] + [illegible] =
$ 4386.1[illegible]
Total cost of [illegible] $ [illegible]
Total cost of [illegible] [illegible] $ [illegible]

Cost of [illegible].

Cost of pile [illegible] $ [illegible] per foot.
= [illegible] = $ [illegible]
Cost of [illegible] [illegible] $ [illegible]
= [illegible] = $ [illegible]
Total cost of [illegible] $ 30[illegible]
Total cost of [illegible] [illegible] =
$1004[illegible] + $ [illegible] = $[illegible]

Estimate of Cost of Superstructure.

Weight of Steel in 75'- 0" Span.

The following weights are for one half of one truss.

Member.	Length	Weight.
Upper Chord.		
2 angles 6" x 4" x 5/8"	30'-0"	1200.00 #
1 Cov. Pl. 12" x 5/8"	30'-0"	612.00
1 Splice Pl. 12" x 5/8"	3'-0"	76.50
1 Splice Pl. 12" x ½"	1'-0"	20.40
End Post.		
2 angles 6" x 4" x 5/8"	10'-0"	400.00
1 Cov. Pl. 12" x ½"	10'-0"	204.00
Lower Chord.		
2 angles 6" x 4" x 7/8"	37'-6"	2040.00
1 Splice Pl. 12" x 3/8"	0'-10"	13.75
1 Splice Pl. 12" x 3/8"	3'-1"	47.17
1 Splice Pl. 12" x 3/8"	2'-6"	37.80
Verticals.		
4 angles 4" x 3" x 5/16"	6'-6"	187.00
4 Batten Pl. 12" x ½"	0'-5"	34.00
4 Ext. Ang. 4" x 3" x5/16"	3'-7"	103.00

Diagonals	Length	Weight.
2 angles 4" x 3" x 11/16"	[illegible]'-[illegible]"	[illegible].40
1 angle 4" x 3½" x [illegible]"	[illegible]'-0"	[illegible].[illegible]
2 angles 3" x 3" x ½"	[illegible]'-[illegible]"	[illegible]
1 clip ang. [illegible]" x [illegible]½" x [illegible]"	[illegible]'-[illegible]"	[illegible].60
4 batten pl. [illegible]" x ¼"	[illegible]'-[illegible]"	[illegible]
4 batten pl. 10" x ¼"	[illegible]'-10"	[illegible]
Gusset Plates.		
[illegible] Plates ½" x [illegible]'-10"	[illegible]	[illegible]
2 Plates ½" x [illegible]'-0"	[illegible]	[illegible]
[illegible] Plates ½" x [illegible]'-[illegible]"	[illegible]	[illegible]
3 Plates ½" x [illegible]'-[illegible]"	[illegible]	[illegible]
2 Plates ½" x 1'-10"	[illegible]	[illegible]
2 Plates ½" x 1'-[illegible]"	[illegible]	[illegible]
Shoe.		
2 angles [illegible]" x [illegible]½" x ½"	[illegible]	[illegible]
2 plates ½" x 15"	[illegible]'-[illegible]"	[illegible].40
Lacing.		
65 @ 1½" x 7/16"	2'-[illegible]"	[illegible].00
18 @ 2" x ¼"	[illegible]	[illegible].62
[illegible] @ 1¼" x ¼"	[illegible]'-2"	[illegible]
70 @ 1¼" x ¼"	[illegible]	[illegible]

[illegible]

[illegible] $+$ [illegible] $=$ [illegible].30 [illegible]

weight of [illegible]

[illegible] x 1001.[illegible] $=$ [illegible], the period of [illegible] -

[illegible] $= \frac{[illegible]}{[illegible]} =$ [illegible]

Weight of [illegible] Span.

Upper Chord	Length	Weight.
[illegible] angles [illegible]" x 4" x $\frac{1}{2}$"	[illegible]	[illegible]
1 Cov. Pl. [illegible]" x [illegible]/[illegible]"	[illegible]'-[illegible]"	[illegible].00
End Post.		
[illegible] angles [illegible]" x 4" x $\frac{1}{2}$"	[illegible]	[illegible]
1 Cov. Pl. [illegible]" x [illegible]"	[illegible]	[illegible]
Verticals.		
[illegible] angles [illegible]" x [illegible]" x [illegible]/[illegible]"	[illegible]	[illegible]
Lower Chord.		
[illegible] angles [illegible]" x 4" x $\frac{1}{2}$"	[illegible]'-[illegible]"	[illegible]
Diagonals.		
[illegible] angles [illegible]" x [illegible]" x [illegible]/[illegible]"	[illegible]'-[illegible]"	[illegible]
4 angles [illegible]" x [illegible]" x $\frac{1}{2}$"	[illegible]'-[illegible]"	[illegible]
Lower Laterals.		
[illegible] angles [illegible]$\frac{1}{2}$" x [illegible]" x [illegible]/[illegible]"	[illegible]'-[illegible]"	[illegible].[illegible]
[illegible] [illegible], [illegible]$\frac{1}{2}$" x [illegible] x $\frac{1}{4}$"	[illegible]'-[illegible]"	[illegible]
1 angle [illegible]$\frac{1}{2}$" x [illegible]$\frac{1}{2}$" x [illegible]"	[illegible]	[illegible]

[illegible]

Stringers. [illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible] $=$ [illegible]

[illegible]

[illegible]

[illegible] $=$ [illegible]

[illegible] $+$ [illegible] $=$ [illegible] weight of the bridge.

[illegible] equal [illegible]

[illegible] $=$ [illegible] weight of steel in [illegible] equal [illegible] $+$ [illegible] $=$ [illegible].

[illegible] $=$ [illegible] painting.

[illegible] it is to be [illegible] will be short. The total [illegible] will [illegible] 50.00 making [illegible] total of [illegible].

[illegible]

Cost of Asphalt Pavement.

There are 136 square yards of pavement to lay on the bridge and the two approaches. The average price per square yard of asphalt pavement is $ 2.17. This is for pavement guaranteed for five years, with a 2" concrete base. 136 x 2.17 = $ 295. total cost of pavement.

Cost on Approaches.

The two approaches will be dirt fill with slopes of 1½ to 1. The total number of cubic yards in the two fills is 1157. There is no over haul and therefore the fill will cost $ .17 per cubic yard. 1157 x .17 = $ 196.70. It will be necessary, to build a wire fence along the approach, the total length being 500 feet. Use of cedar posts 7" in diameter with the wire at 15" centers. This will cost about $ [illegible].

Total cost of superstructure = $ [illegible].

Total cost of whole structure = [illegible]

[illegible]

The total cost is $[illegible].

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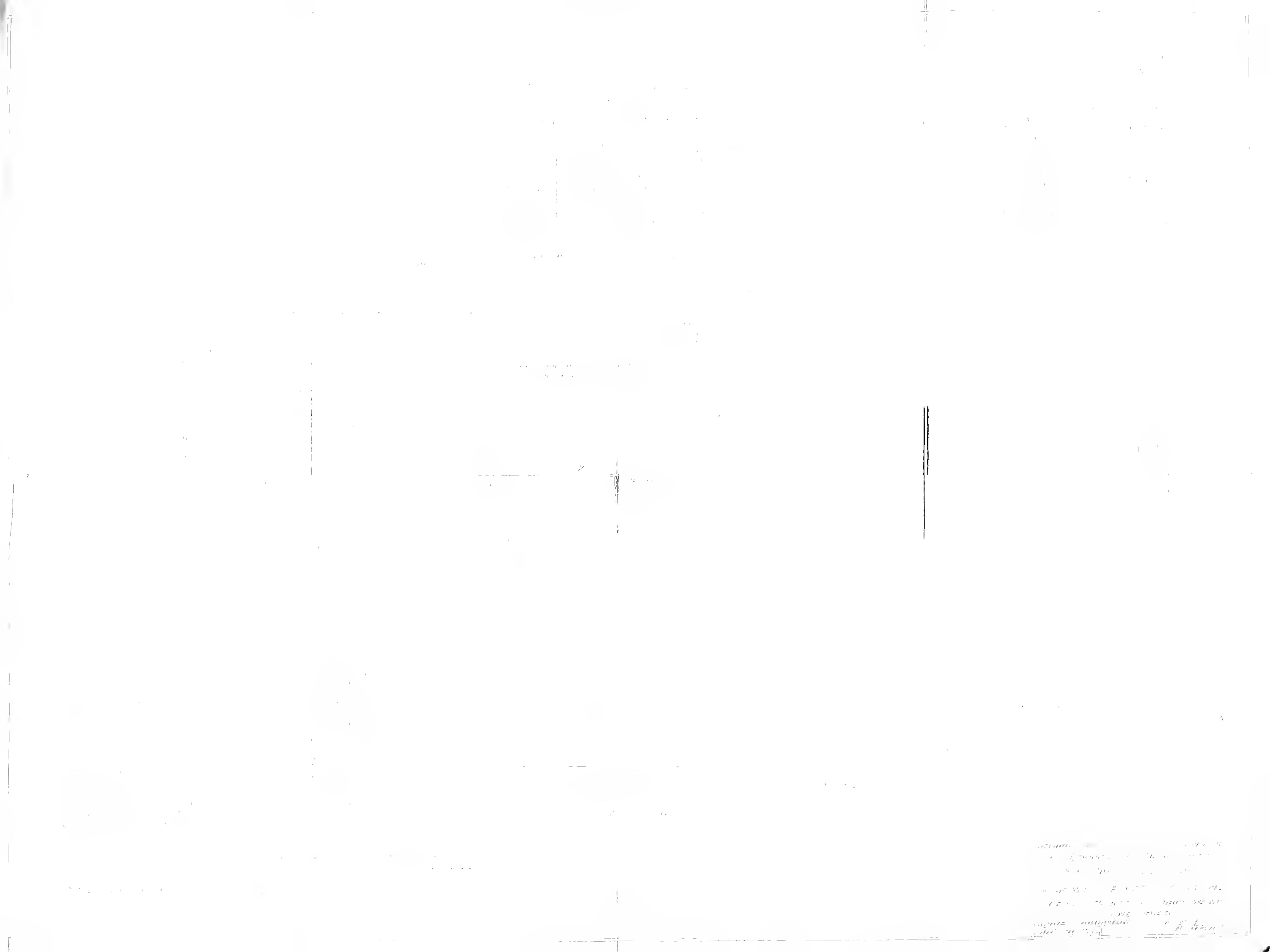

[illegible]